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SEISMIC PERFORMANCE OF AN INSTRUMENTED TEN-STOREY REINFORCED CONCRETE BUILDING

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SUMMARY

Results from analytical studies conducted on an instrumented ten-storey reinforced concrete building which experienced ground accelerations in excess of 0·6*g* during the 1987 Whittier-Narrows California earthquake and suffered only minimal damage are presented. Using the dynamic characteristics inferred from accelerations recorded in the building during the earthquake, a mathematical model was calibrated to study the response of the building and to explain its good behaviour despite the apparent severity of the motions recorded in the basement of the building. Very good correlation was obtained between the computed and recorded response of the building. Non-linear analyses were conducted to evaluate the strength and deformation capacity of the building and to estimate its response in the event of more severe earthquake ground motions. Special emphasis is given to the evaluation of the overstrength of the building. Lateral overstrengths larger than 4·2 and larger than 5·7 were computed for the longitudinal and transverse directions of the building, respectively. It is concluded that these high levels overstrength in the building played an important role in limiting the damage during the Whittier-Narrow earthquake. Since the estimation of inelastic deformations during severe earthquake ground motions depends on the actual strength of the building, it is recommended to consider explicitly probable values of this overstrength in the strength reduction factors.

KEY WORDS: reinforced concrete buildings; seismic response; system identification; overstrength

INTRODUCTION

When subjected to moderate or intense earthquake ground motions well-instrumented and fully documented buildings offer an excellent opportunity to conduct detailed analyses to study their seismic response and to evaluate the effectiveness of analytical techniques to estimate their seismic vulnerability.

This paper presents analytical studies conducted on an instrumented ten-storey reinforced concrete building. The building, which was designed according to 'pre-San Fernando earthquake' seismic design provisions, experienced ground accelerations in excess of 0·6*g* during the 1987 Whittier-Narrows California earthquake and suffered only minimal damage.

The objectives of this paper are threefold: (i) To try to explain the absence of a larger structural damage by computing the response of the building when subjected to the Whittier-Narrows earthquake using an analytical model calibrated to approximately match the dynamic characteristics of the building inferred from records obtained in the building during the earthquake; (ii) To compare the computed response with that recorded during the earthquake in order to evaluate the effectiveness of presently available analytical techniques to estimate the response of the buildings to moderate earthquake ground motions; and (iii) To conduct non-linear static and dynamic (time-history) analyses of the building to evaluate its lateral strength and deformation capacities as well as to estimate the response of the building under more severe ground motions.

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The study presented herein is part of an ongoing research program at the Earthquake Engineering Research Center of the University of California at Berkeley whose ultimate objective is the development of reliable methods for the design of new buildings and aiding in the vulnerability assessment of existing buildings and the selection of efficient strategies and techniques for the seismic upgrading of existing hazardous buildings.

DESCRIPTION OF THE BUILDING

The ten-storey building studied herein is a reinforced concrete structure designed and constructed in 1972. It is located at latitude 33·98°N and longitude 118·04°W within the Los Angeles metropolitan area.

In the longitudinal (north–south) direction the structural system of the building is a moment-resisting frame consisting of two external frames designed to carry most of the lateral loads and two interior frames designed primarily to carry vertical loads. Figure 1 shows a typical floor plan of the building. The exterior frames consist of 50·8 cm by 50·8 cm reinforced concrete columns and 61 cm × 61 cm beams. The interior frames consist of 40·6 cm × 40·6 cm columns and a cast-in-place 16·5 cm thick concrete slab (flat plate).

The structural system in the transverse (east–west) direction is a dual system composed of reinforced-concrete coupled shear walls in the north and south ends of the building, two smaller shear walls surrounding the elevators and flexible frames (columns and flat plate).

The soil conditions at the site consist of quaternary alluvium deposits of medium-grain sand sediments. The foundation of the building consists of spread footings. The height of the building is 27·4 m above the ground level. Figure 2 shows an elevation of the transverse direction of the building. Interstorey heights are 3·66 m in the first storey and 2·64 m for the second through tenth stories.

Longitudinal and transverse reinforcement details of typical columns are shown in Figure 3. Reinforcement details of typical beams are shown in Figure 4. Transverse reinforcement in the columns consists of square ties with 90° hooks at the corners. Similarly, ties in the beams are not closed, instead consisting of U-shaped ties with alternating caps. It is worth mentioning that the amount of specified transverse reinforcement in critical regions of columns and beams exceeds minimum code requirements at the time of construction. However, by today's standards the amount of transverse reinforcement and the type of detailing would be considered as inadequate.

The specified 28-day strength of the concrete (f'_c) was 27·6 MPa (4000 psi) for the first floor and columns up to the 6th floor, and 21·7 MPa (3000 psi) for the rest of the structure. All reinforcing steel was grade 60 ($f_y = 413{\cdot}8$ MPa) conforming to ASTM specification A-615.

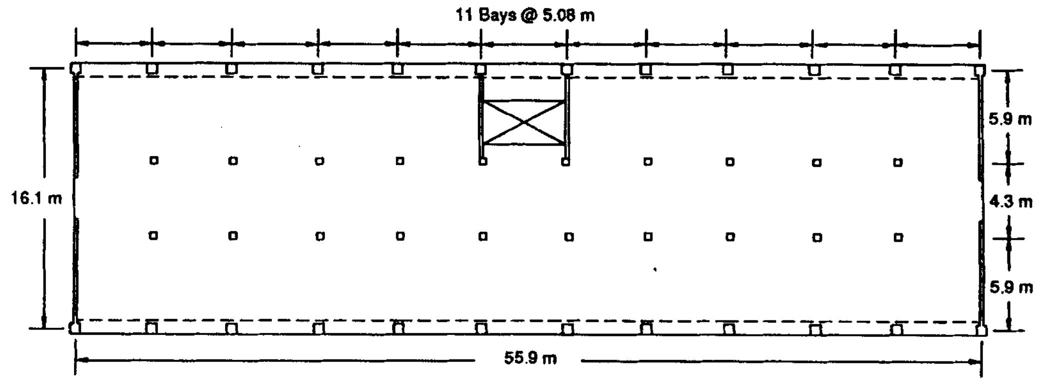

Figure 1. Typical floor plan of the building

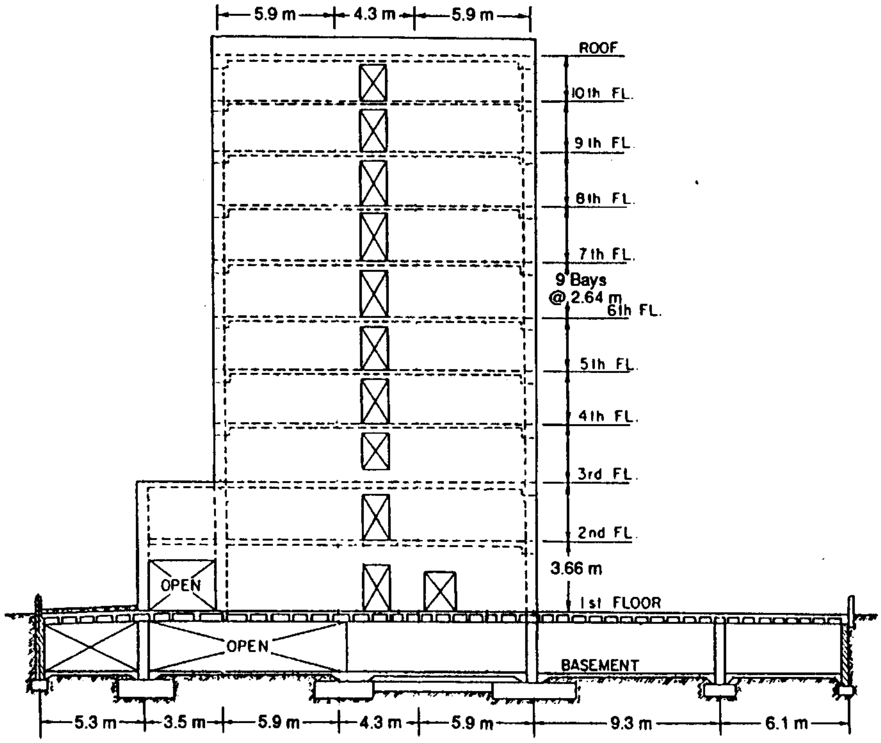

Figure 2. Elevation of the transverse direction of the building

SEISMIC DESIGN CRITERIA

The building was designed and constructed in 1972 according to the 1970 edition of the Uniform Building Code.[1] Under these design recommendations the building had to be designed to resist the lateral forces given by the following expression:

$$V = ZKCW \tag{1}$$

where V is the total horizontal force to be resisted, Z is a factor which depends on the seismic zone, K is a factor that is a function of the structural system of the building, W is the total dead load of the building, and C is given by

$$C = \frac{0{\cdot}05}{T^{1/3}} \tag{2}$$

where T is the fundamental period of the building which is given by

$$T = \frac{0{\cdot}05h_n}{D^{1/2}} \tag{3}$$

where h_n is the height of the roof above the ground level (in feet), and D is the dimension of the building (in feet) parallel to the applied forces. According to this set of specifications equation (2) was to be used for all buildings with exception of buildings where 100 per cent of the lateral forces were carried by moment-resisting frames, in which case the period could be estimated by

$$T = 0{\cdot}1N \tag{4}$$

where N is the number of stories.

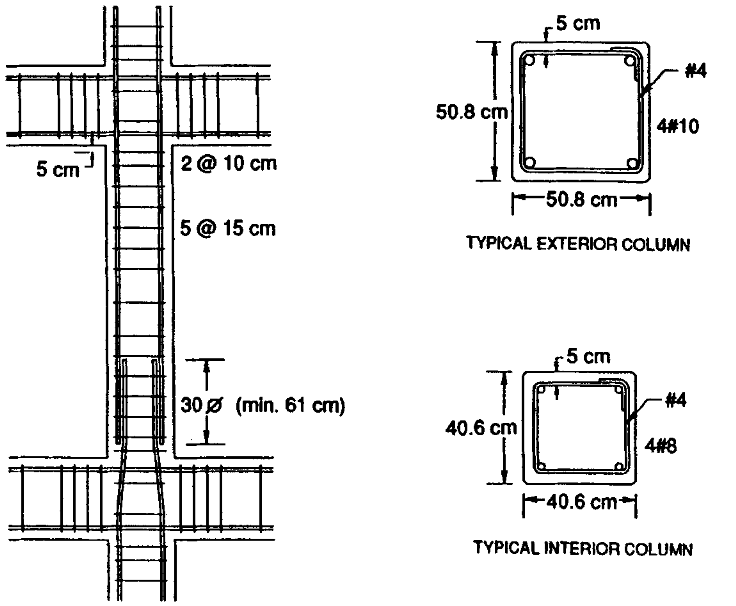

Figure 3. Reinforcing details in the columns

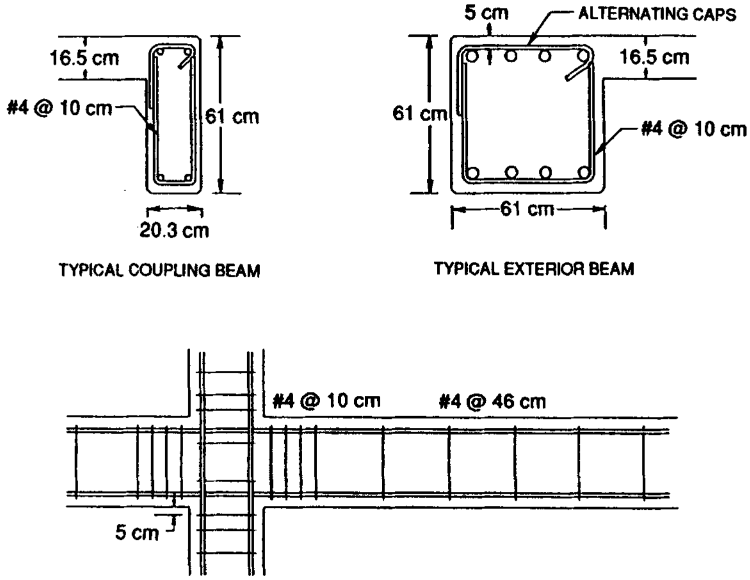

Figure 4. Reinforcing details in the beams

When applied to this building, equation (3) leads to an unrealistically low period of 0·33 s for the longitudinal (north–south) direction, while for the transverse (east-west) direction the same expression leads to a very reasonable approximation of 0·62 s. For the longitudinal direction equation (4) predicts a more realistic fundamental period (1·0 s).

For the transverse (dual system) direction the use of equations (1)–(3) along with corresponding load and strength reduction factors lead to the following minimum code-expected lateral strength:

$$\frac{V}{W} = \frac{(0{\cdot}75 \times 1{\cdot}7 \times 1{\cdot}1)ZKC}{0{\cdot}9} = \frac{(0{\cdot}75 \times 1{\cdot}7 \times 1{\cdot}1)(0{\cdot}8)(0{\cdot}586)}{0{\cdot}9} = 0{\cdot}073 \tag{5}$$

Table I. Peak accelerations (in g's) recorded in the building during three earthquakes

Floor	Direction	1/1/76 event ($M_L = 4{\cdot}2$)	1/10/87 event ($M_L = 5{\cdot}9$)	4/10/87 event ($M_L = 5{\cdot}3$)
10th Floor	90° (A)	0·18	0·53	0·43
5th Floor	90°	0·23	0·62	0·58
Basement	90°	0·16	0·60	0·33
10th Floor	180° (B)	0·04	0·43	0·17
5th Floor	180°	0·06	0·55	0·30
Basement	180°	0·06	0·40	0·30

Notes: (A) Dual system (shear walls & moment-resisting frame)
(B) Moment-resisting frame

For the longitudinal (moment-resting frame) direction, equations (1), (2), and (4) and corresponding load and strength reduction factors lead to a minimum code-expected lateral strength of

$$\frac{V}{W} = \frac{(0{\cdot}75 \times 1{\cdot}7 \times 1{\cdot}1)ZKC}{0{\cdot}9} = \frac{(0{\cdot}75 \times 1{\cdot}7 \times 1{\cdot}1)(0{\cdot}67)(0{\cdot}05)}{0{\cdot}9} = 0{\cdot}052 \tag{6}$$

INSTRUMENTATION OF THE BUILDING

The building forms part of the National Strong-Motion Instrumentation Network (NSMIN) operated by the U.S. Geological Survey (USGS). The building instrumentation consists of three SMA-1 analog accelerographs (each capable of recording three components of motion) located in the south end of the basement, 5th floor and 10th floor.

The first set of earthquake records obtained from the building were obtained on the 1st January 1976 Whittier earthquake. The epicenter of this magnitude 4·2 (M_L) earthquake was located approximately 13·8 km (8·6 miles) east of the building.[2] In the transverse (east–west) direction peak accelerations of 0·16g, 0·23g, and 0·18g were recorded at the basement, 5th floor, and 10th floor, respectively. In the longitudinal (north–south) direction peak accelerations of 0·06g, 0·06g, and 0·04g were recorded at the basement, 5th floor, and 10th floor, respectively.

At the time of writing, the largest accelerations recorded in the building correspond to those measured during the 1st October 1987 Whittier-Narrows earthquake. The epicenter of this magnitude 5·9 (M_L) earthquake was approximately 10 km (6·2 miles) north of the building. Among more than 250 strong-motions accelerograph stations (operated by the USGS, the California Strong Motion Instrumentation Program, and the University of Southern California), that were triggered in this earthquake, the largest peak ground acceleration was recorded in the basement of this building. In the transverse (east–west) direction peak accelerations of 0·63g, 0·62g, and 0·53g were recorded in the basement, 5th floor, and 10th floor, respectively. In the longitudinal (north–south) direction peak accelerations of 0·43g, 0·55g, and 0·40g were obtained in the basement, 5th floor, and 10th floor, respectively.[3]

A magnitude 5·3 (M_L) aftershock of the Whittier-Narrows earthquake which occurred on 4 October 1987 produced peak accelerations of 0·33g, 0·58g, and 0·43g in the transverse direction in the basement, 5th floor and 10th floor, respectively. Recorded peak accelerations in the longitudinal direction were 0·30g, 0·30g, and 0·17g in the basement, 5th floor, and 10th floor, respectively.[4]

Peak accelerations recorded in the building during these three earthquakes are summarized in Table I. Two general trends can be observed from this table: (a) the largest acceleration has always been recorded at the 5th floor and not the 10th floor, indicating a possible strong participation of the second mode; and (b) larger accelerations have been recorded in the transverse (stiff) direction than in the longitudinal (flexible) direction.

Table II. Peak responses in the building during the 1st October 1987 Whittier-Narrows earthquake

Floor	Direction	Max. rel. velocity (cm/s)	Max. rel. displ. (cm)
10th	90° (A)	55·87	2·94
5th	90°	22·07	1·18
10th	180° (B)	46·04	7·14
5th	180°	39·55	4·11

Notes: (A) Dual system (shear walls & moment-resisting frame)
(B) Moment-resisting frame

Besides the earthquakes listed in Table I, instruments in the building were also triggered during the 8 July 1986 North Palm Springs, California earthquake ($M_L = 5{\cdot}9$), the 28 February 1990 Upland, California earthquake ($M_L = 5{\cdot}5$), and the 28 June 1992 Landers, California earthquake ($M_L = 7{\cdot}4$). Maximum accelerations recorded in the building during these events were smaller than 10 per cent of acceleration due to gravity.[5]

RESPONSE DURING THE 1987 WHITTIER-NARROWS EARTHQUAKE

The 1st October 1987 Whittier-Narrows earthquake ($M_L = 5{\cdot}9$) had its epicenter about 15 km (9·3 miles) northeast of downtown Los Angeles, at a focal depth of about 14 km (8·7 miles).

Major damage occurred within 5 km (3 miles) of the building, including several partial collapses in the Whittier downtown shopping area (Whittier Village). A Modified Mercalli Intensity (MMI) of VII was assigned to the area where the building is located.[6] No damage was found in the building during two different post-earthquake visual inspections.[7,8]

Motions recorded in the transverse direction of the building have a larger intensity than those recorded in the longitudinal direction. Although the duration of the digitized records is 30 s, using Trifunac's rule[9] the duration of the strong motion is only about 4 s. Peak acceleration values are summarized in Table 1. Relative (with respect to the base) velocity and displacement time histories at the 5th and 10th floor were computed by numerical integration, high-pass filtering (using a Ormsby digital filter with a linear decay between a cut-off frequency 0·20 Hz and a roll-off termination frequency of 0·18 Hz), and base-line correction of recorded accelerations. Peak values are summarized in Table II.

The occurrence of a moderate or strong earthquake can be viewed as a full-scale experiment on a structure, and if the structural motion is recorded, it offers the opportunity to make a quantitative study of the structure at dynamic force and deflection levels directly relevant to earthquake-resistant design. A particularly important aspect of this quantitative study is the determination of the dynamic characteristics of the structure, which can be estimated from recorded accelerations through system identification techniques.

In this study the following frequency-domain system identification techniques were used: (i) non-parametric, time-invariant; (ii) non-parametric, time-variant (moving-window Fourier analysis); and (iii) parametric, time-invariant.

In the first technique the structure is idealized by a non-parametric, time-invariant linear model in which the input $x(t)$ and output $y(t)$ are related in the frequency domain through the following input–output relation

$$Y(\omega) = H(\mathrm{i}\omega)\, X(\omega) \tag{7}$$

where $X(\omega)$ and $Y(\omega)$ are the Fourier transform of the input and output time-series, and $H(\mathrm{i}\omega)$ is the transfer function which corresponds to the Fourier transform of an impulse response function $h(t)$.

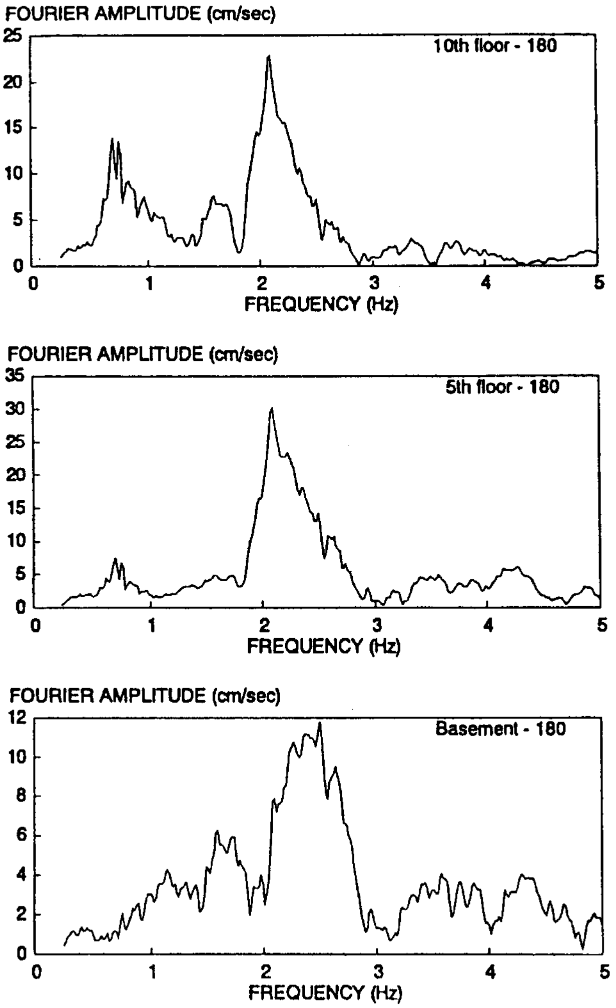

Figure 5. Fourier amplitude spectra of the accelerations recorded in the longitudinal direction of the building

The second system identification technique (non-parametric, time-variant) is essentially the same as the approach just described, except that in order to identify the variation of parameters in time, a window smaller than the total duration of the record, is 'moved' in time. The size of the window is typically selected to be a function of the fundamental period of the building. In this study, a window with a duration of approximately five times the fundamental period of the building was used.

The parametric time-invariant technique used herein consists in computation a set of parameters of a simple mathematical model of the structure which minimizes the difference between the smoothed Fourier transform of the recorded acceleration time histories and the Fourier transform of the computed response.

Despite some limitations, these techniques have successfully been used in the past to identify the dynamic characteristics of buildings that have been excited by earthquake ground motions.[10,11] For a more detailed description of these techniques the reader is referred to References 11 and 12.

Fourier amplitude spectra of horizontal acceleration records in the longitudinal direction of the building are shown in Figure 5. It can be seen that the ground has its strongest input in a band between 2 and 3 Hz (predominant period between 0·33 and 0·5 s). Motion in this frequency band is amplified in the structure, particularly at the 5th floor. Figure 6 shows Fourier amplitude spectra of horizontal accelerations recorded

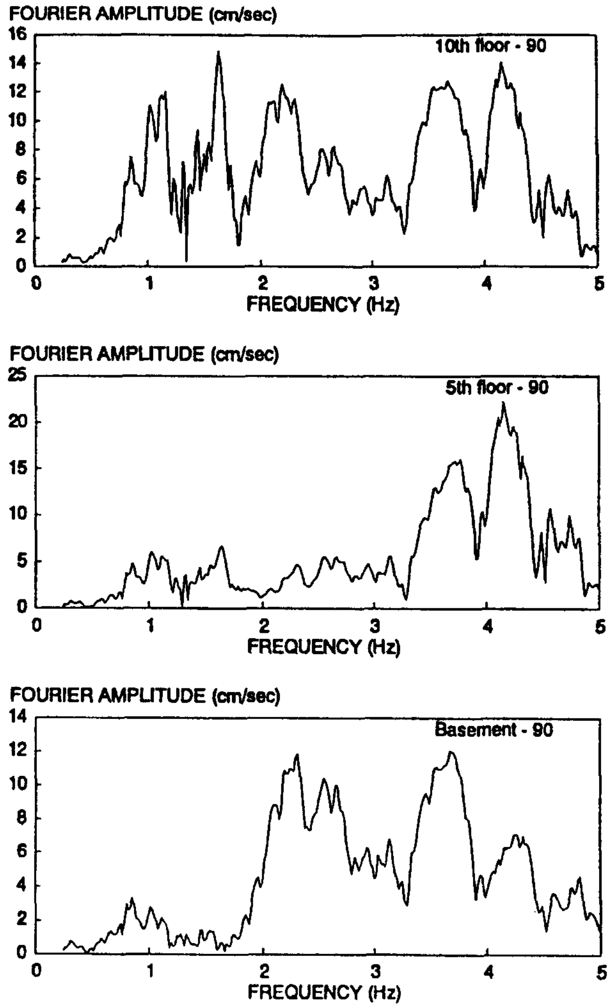

Figure 6. Fourier amplitude spectra of the accelerations recorded in the transverse direction of the building

in the transverse direction of the building. The input motion (basement) is particularly strong for frequencies between 2 and 3 and around 3·5 Hz.

Transfer functions for the longitudinal motions (moment-resisting frame) direction are shown in Figure 7. From this figure, 1st, 2nd, 3rd and 4th mode frequencies are noticeable at 0·70, 2·06, 3·18 and 4·09 Hz, respectively. The ratios between these frequencies ($f_2/f_1 = 2{\cdot}9, f_3/f_1 = 4{\cdot}6, f_4/f_1 = 5{\cdot}9$) agrees reasonably with what could be expected for moment-resisting frame buildings.[13] Transfer functions for the horizontal accelerations recorded in the transverse (dual system) direction of the building are also shown in Figure 7. The largest amplitude peak (at 1·64 Hz) corresponds to the fundamental mode. The second peak (at 4·1 Hz) and the vibrational mode shape associated with it, corresponds to the second mode. The ratio of second to first modal frequencies is 2·5, which is low compared to what could be expected for the structural system in this direction. Similar modal frequencies were obtained with the first and third identification techniques. A summary of periods of vibration identified from the recorded earthquake motions is presented in Table III.

Since the building has only one instrument on each floor, torsional modes could not be reliably identified. Computed damping ratios varied depending on the resolution, smoothing, and filter used in computing the Fourier amplitude spectra. For the first mode the damping varied between 4·9 and 7·3 per cent for the longitudinal direction and between 3·3 and 4·2 per cent for the transverse direction.

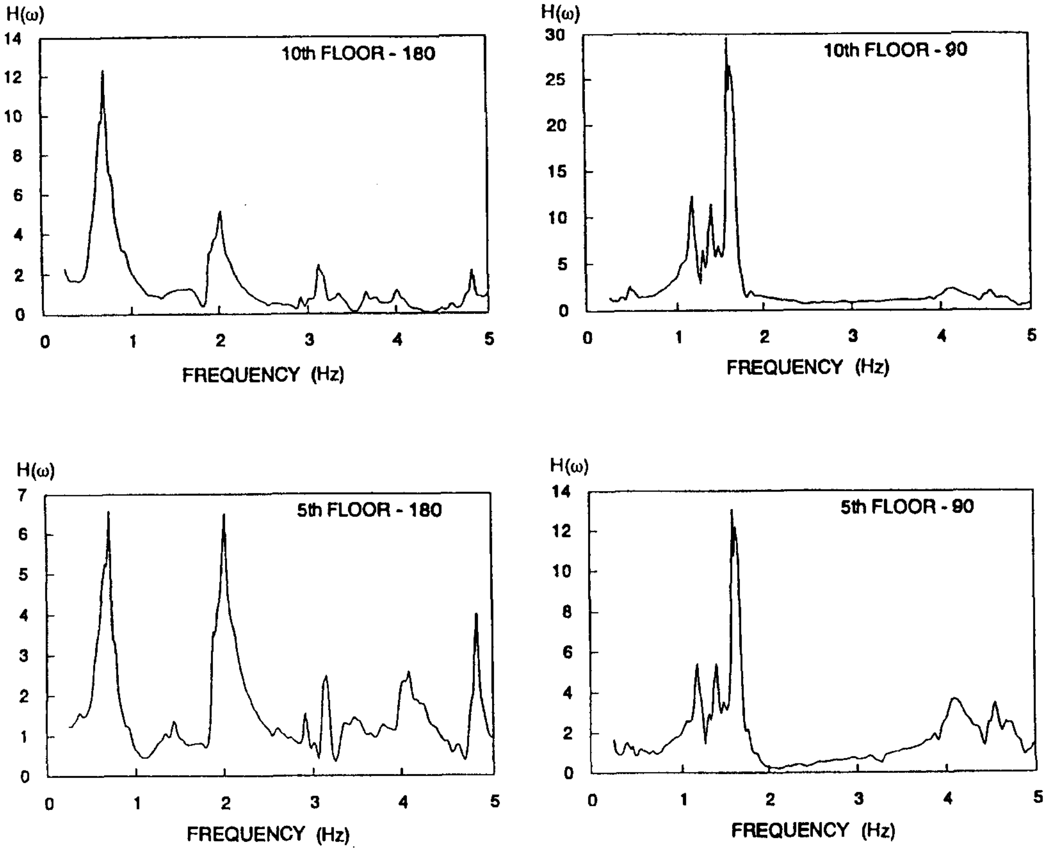

Figure 7. Transverse functions of accelerations recorded in building

Table III. Translational periods of vibration identified from acceleration records from the 1st October 1987 Whittier-Narrows earthquake

Direction		Periods of vibration [sec] 1st mode	2nd mode	3rd mode
East–West	90° (A)	0·61	0·24	0·09
North–South	180° (B)	1·43	0·48	0·31

Notes: (A) Dual system (shear walls & moment-resisting frame)
(B) Moment-resisting frame

Moving-window Fourier analyses were conducted using windows of 5 s for the transverse direction and 7·5 s for the longitudinal direction, moving at 2·5 s intervals. An increase of 4·2 per cent in the first translational period was observed in the transverse direction between the 2·5 and 5·0 s marks. No change in fundamental period was observed in the longitudinal direction.

The response of the building during the 1st October 1987 Whittier-Narrows earthquake was studied by conducting a three-dimensional linear elastic time history analysis. Special emphasis was placed on studying the effectiveness of this kind of analysis at capturing the response of the building when subjected to moderate earthquake ground motions.

Table IV. Participating mass as a percentage of the total mass for the first two modes of the structure

Direction	Participating mass		
	1st mode	2nd mode	1st & 2nd modes
East–West 90° (A)	62·4	19·5	81·9
North–South 180° (B)	75·3	13·4	88·8

Notes: (A) Dual system (shear walls & moment-resisting frame)
(B) Moment-resisting frame

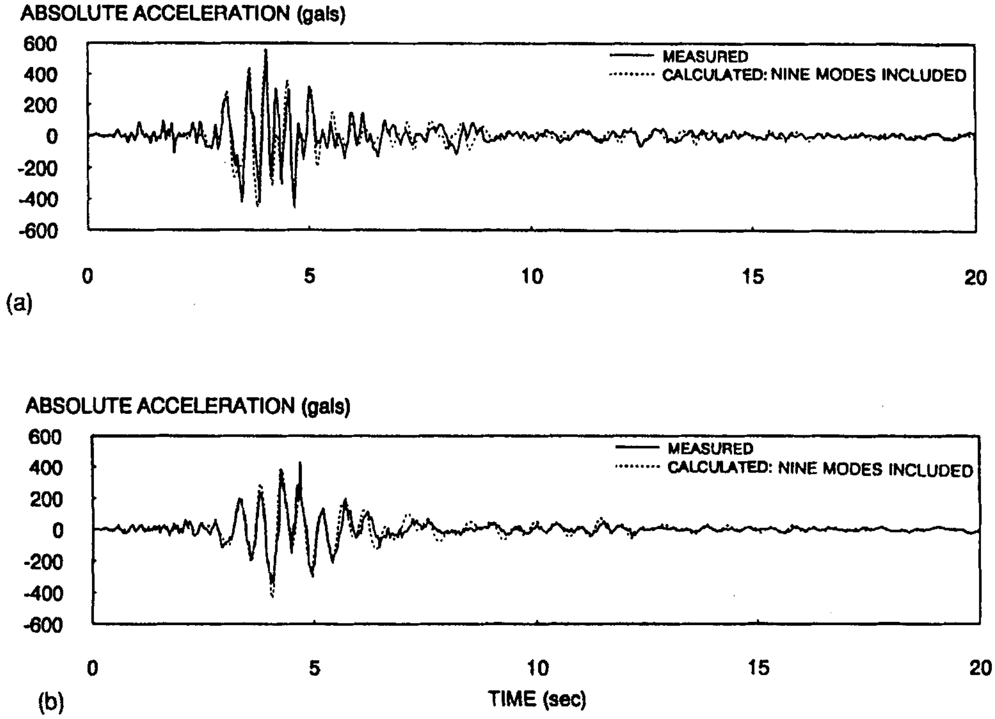

Figure 8. Comparison of measured and calculated acceleration time histories for the 10th floor: (a) Transverse direction; (b) Longitudinal direction

The building was modelled as a three-dimensional array of linear elements. The floor diaphragm was assumed to be rigid and the columns and shear walls to have fixed bases. Cracked transformed sections were used for all structural elements, and the joints were assumed to be rigid over 75 per cent of their dimensions. The complete model consisted of 899 nodes, 1642 members, and 2514 degrees of freedom.

An eigenvalue analysis was conducted in order to calibrate the model such that the fundamental periods (in both directions) agreed with those identified from earthquake records. Keeping the computed masses constant, adjustments in member stiffnesses of less than 7 per cent were necessary to calibrate the model. Table IV summarizes the participation of modal masses of the first two in each direction, as a percentage of the total mass. For both directions, the first and second modal masses account for more than 80 per cent of the total mass.

Time-history analyses were conducted using as input the acceleration time histories recorded in the basement of the building during the Whittier-Narrows earthquake. A time increment of 0·02 s and damping ratio of 5 per cent (for all modes) were used in the analyses. Figure 8 compares measured and calculated

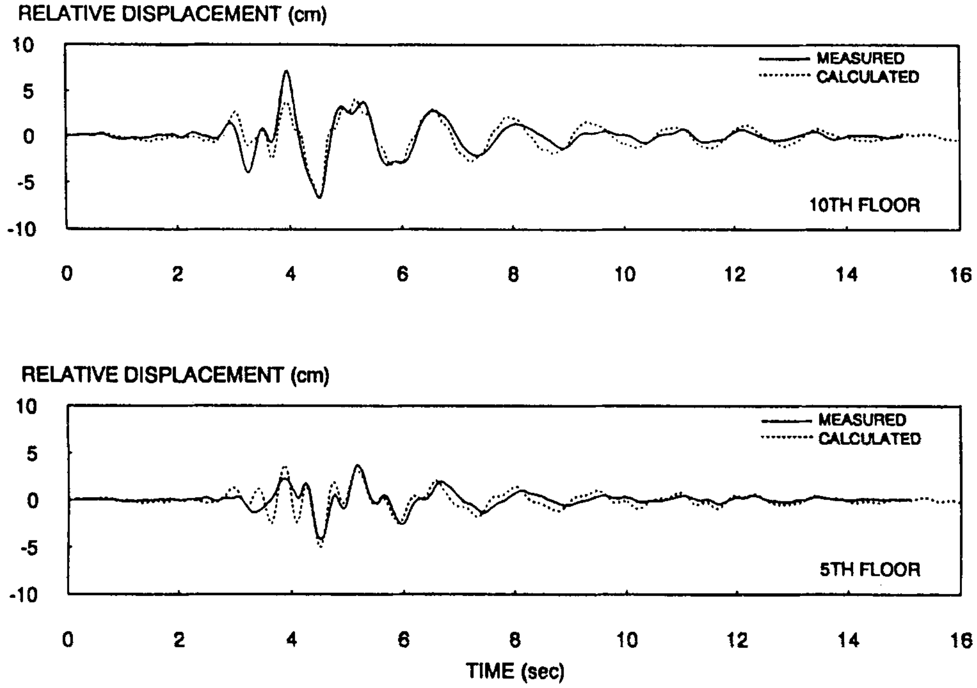

Figure 9. Comparison of measured and calculated displacement time histories for the longitudinal direction of the building

acceleration time histories in the 10th floor. As shown in this figure, the correlation between measured and calculated response is, in general, very good. The contribution of the second translational mode in each direction to the total acceleration is important, particularly in the longitudinal direction which explains why maximum recorded accelerations have typically been larger in the 5th floor than in the 10th floor. Examination of the Fourier amplitude spectrum at the base of the buildings (i.e., input motion) in the longitudinal directions reveals that the ground motion has much higher ordinates for frequencies near the second mode (2·06 Hz) than for the fundamental frequency (0·70 Hz). Similarly, the Fourier amplitude spectrum at the base of the building in the transverse directions shows that the ground motion has much higher ordinates for frequencies near the second mode (4·1 Hz) than for the first mode frequency (1·64 Hz).

Comparisons of calculated and measured displacements for the longitudinal and transverse directions of the building are shown in Figures 9 and 10, respectively. Agreement between computed and inferred-from-recorded data is better for the longitudinal direction than for the transverse direction where the linear elastic (time-invariant) model is unable to capture the lengthening of the fundamental period in the transverse direction that occurs approximately 4 s after the start of the earthquake. When comparing displacements time histories, one must consider that differences are due not only to imperfect modelling assumptions but also due to uncertainties in the computation of relative displacement (with respect to the base) time histories from recorded acceleration time histories. The type of filter, and especially the selection of corner frequencies for the high-pass digital filter may produce errors in the estimation of displacements inferred from strong-motion accelerograms.[14] Selection of the type of filter and the cut-off frequency depends of the level of noise in the record which depends on the type of instrument.[15]

A summary of the results from the time-history analyses and a comparison with recorded data and data inferred from recorded data is presented in Table V. The correlation attained by the linear elastic model is very good considering the uncertainties associated with the estimation of mechanical properties of the structural members, modelling assumptions, estimation of the masses, uncertainties associated with the structural damping, etc.

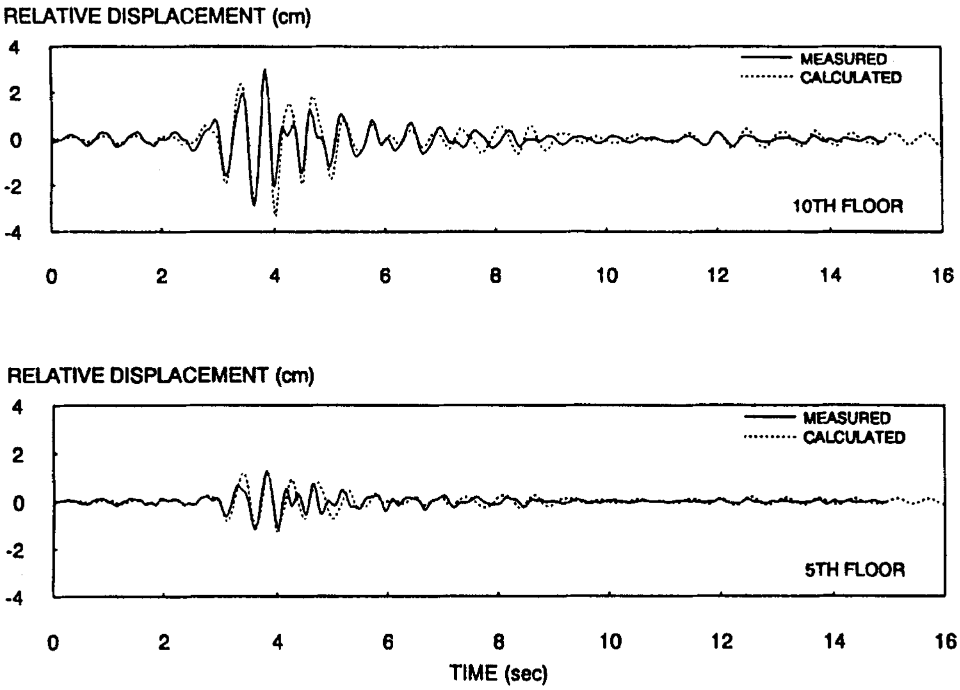

Figure 10. Comparison of measured and calculated displacement time histories for the transverse direction of the building

Table V. Comparison of computed and measured responses of the building during the Whittier-Narrows earthquake

Max. response parameter	Direction	Floor	Computed	Measured
Acceleration [cm/sec^2]	90° (A)	10th	536·3	523·7
		5th	587·9	612·2
	180° (B)	10th	418·3	425·8
		5th	528·4	542·8
Rel. velocity [cm/sec]	90°	10th	54·70	55·87
		5th	21·10	22·07
	180°	10th	46·55	46·04
		5th	40·82	39·55
Rel. displacement [cm]	90°	10th	2·92	2·94
		5th	1·22	1·18
		10th	7·12	7·14
	180°	5th	4·33	4·11

Notes: (A) Dual system (shear walls & moment-resisting frame)
(B) Moment-resisting frame

The maximum computed interstorey drift index for the longitudinal direction of the building (moment-resisting frame) was 0·0034, which occurred at the fourth storey. For the transverse direction, the maximum computed interstorey drift index was 0·0021. The maximum base shears resulting from the time-history analyses are 6780 kN (1524 kips) and 16094 kN (3618 kips) for the longitudinal and transverse directions, respectively. Normalized base shears with respect to the total weight are 9·9 and 23·6 per cent for

the longitudinal and transverse directions, respectively. Thus, the maximum base shear experienced in both directions during this moderate-magnitude earthquake exceeded the minimum required by the 1970 UBC.

EVALUATION OF THE SUPPLIED CAPACITIES TO THE BUILDING

One of the most important aspects of the seismic evaluation of existing structures is the assessment of their lateral strength and stiffness, and in particular the estimation of the level of lateral deformations required to initiate damage and the maximum deformation capacity of the structure. For this purpose, a series of nonlinear static-to-collapse two-dimensional analyses were conducted.

Non-linear modelling of structural members was based in section analyses of their critical regions. For this purpose moment–curvature relations were computed for 18 different cross-sections. Additionally, moment–axial load interaction diagrams were computed for 12 different column or shear wall cross-sections. Details of assumed material properties and constitutive models can be found in Reference 12.

Longitudinal (NS) direction

To model the longitudinal direction of the building, the exterior frames were lumped together, assuming a rigid floor diaphragm. Similarly, the interior frames were lumped together. The slab in the interior frames was modelled by equivalent beams. The mathematical non-linear model consisted of 289 nodes, 519 members and 780 degrees of freedom.

Load–deformation relations were determined by imposing assumed shapes for lateral load distributions over the height of the structure and increasing the total load monotonically from zero up to incipient collapse. For this purpose two loading patterns were used, triangular and rectangular (uniform). Figure 11 shows the relationship between the base shear normalized by the weight of the structure and the roof displacement normalized by the height at this level. The uniform loading pattern leads to 24 per cent higher initial stiffness and 19 per cent higher maximum lateral strength that those computed with the triangular loading pattern. The maximum lateral strength of the building is 22 per cent of its weight when subjected to a triangular load distribution and 26 per cent for a uniform loading pattern. The ratio between the maximum base shear and the base shear at first significant yielding is 1·38 for both lateral loading patterns. Structural damage is initiated in both cases at a normalized roof displacement of 0·0038, which corresponds to a maximum interstorey drift index of 0·005 occurring in the fourth storey.

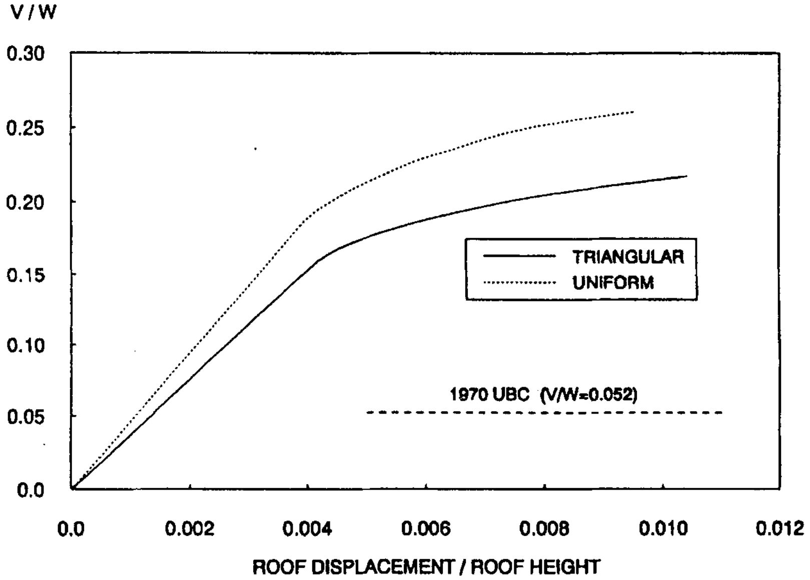

Figure 11. Non-dimensional force–displacement relationships computed for the longitudinal direction of the building

The maximum deformation capacity of the building in the longitudinal direction is expected to be controlled by the available rotation capacity of the columns and by the shear capacity of their corresponding beam-column joints in the 3rd to 7th floors where important concentrations of inelastic deformations occur. For the maximum deformation shown in Figure 11 rotations in excess of 0·016 rad are computed in the fourth-storey columns. While the amount of transverse reinforcement at the end of these columns is, in general, superior to that observed in buildings designed according to 'pre-San Fernando' detailing requirements, the effectiveness of the ties to provide confinement to the concrete core for these levels of deformation is not expected to be good because of the 90° hooks at the corners of the section (see Figure 3). It has been observed that concrete columns with this type of detailing do not exhibit good behaviour, since when the cover concrete cover is lost the end of the tie leg at the 90° hook moves away from the longitudinal bar it engages, resulting in complete loss of anchorage of the tie[16] and therefore there is a loss of confinement and restraining capacity against local buckling of the longitudinal reinforcement. For the maximum deformation shown in Figure 11 joint shear stresses at the fifth floor are 4·51 MPa which are similar to those measured at failure in recently tested beam-column connections that had little or no transverse reinforcement.[17]

Also shown in Figure 11 is the minimum lateral strength required by the 1970 UBC. It can be observed that the building has a lateral strength significantly higher than that for which it was designed. The computed overstrengths of the building are 4·17 and 5·0 for the triangular and uniform loading patterns, respectively.

Despite being subjected to a significant peak ground acceleration during the 1987 Whittier-Narrow earthquake, both the maximum displacement at the tenth floor (7·1 cm) and the maximum interstorey drift index (0·0034) experienced in the building during the earthquake were below the deformation levels required to initiate yielding in the structure, thus explaining the absence of structural damage in the longitudinal direction of the building.

Transverse (EW) direction

Shear walls were modelled as beam-column elements with rigid beams extending from the centre line of the shear wall to each edge. The increase in axial load in the shear walls due to outrigger effects was included by adding non-linear springs with stiffnesses equal to those of the beams framing perpendicularly into the shear walls. A rigid floor diaphragm was also assumed for this direction of the building. The complete mathematical model consisted of 121 nodes, 206 members, and 342 degrees of freedom.

Load–displacement relationships corresponding to triangular and rectangular lateral load patterns are shown in Figure 12. The behaviour of the building is characterized by a gradual loss in lateral stiffness due to yielding of the coupling girders at the ends of the building, followed by a more drastic change in stiffness due to yielding of the shear walls. Strengths corresponding to the first significant yield (i.e., when yielding of the shear walls occurs) of 32 and 43 per cent of the total weight of the building were computed for the triangular and uniform loading patterns, respectively. The yielding of the coupling girders was estimated to initiate for interstorey drift indexes as little as 0·0012. The yielding of shear walls is initiated at normalized roof displacements of 0·0024 and 0·0026 for the triangular and uniform loading patterns, respectively.

The maximum deformation capacity is controlled by the shear and rotation capacity of the shear walls. For the maximum deformation shown in Figure 12, the average shear stresses in the coupled shear walls is 3·76 MPa which could initiate a shear failure of the web, particularly under several large-deformation reversals. Average shear stresses in the north west wall were computed as high as 5·21 MPa because of the presence of a large opening which accommodates the trash room door.

Results from the non-linear static analyses indicate that for the levels of deformation experienced in the transverse direction of the building during the 1987 Whittier-Narrow earthquake, yielding of the coupling girders at the end of the building should have occurred. Due to the apparently different behaviour between the analytical results and the behaviour reported in the surveys of the building (which reported that there was no damage), a new inspection was conducted by the authors. This new inspection revealed that coupling girders in the south end of the building had in fact yielded, while in the coupling girders of the north-end of the building diagonal shear cracking had occurred at all floor levels above the second floor due to the

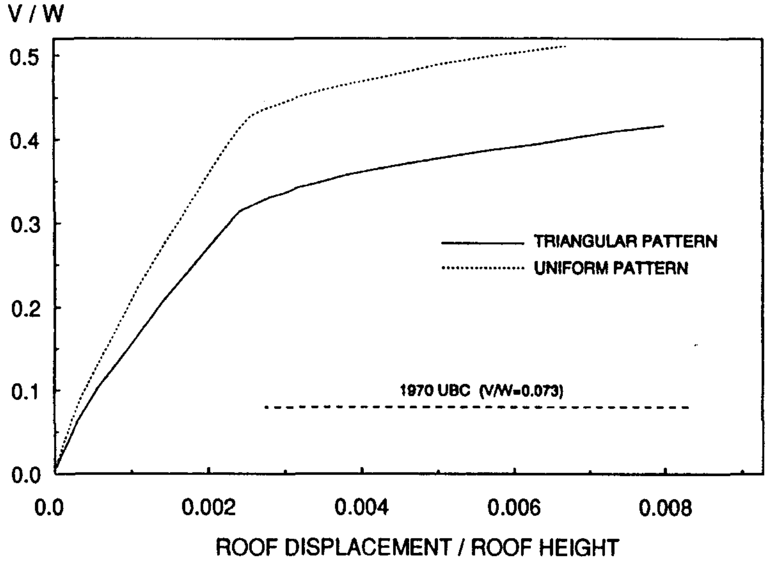

Figure 12. Non-dimensional force–displacement relationships computed for the transverse direction of the building

presence of an opening in the coupling girders which is used for air-conditioning of the building. The yielding and diagonal shear cracking of the coupling girders could explain the small lengthening of the first-mode transverse period of vibration which was inferred from the acceleration time histories recorded in the building during the earthquake.

PERFORMANCE UNDER MORE SEVERE EARTHQUAKE GROUND MOTIONS

The prediction of damage to a specific building through non-linear time-history analysis requires the availability of the full characterization of the acceleration time history of the ground motion that will drive the structure to its critical response, selected from all the possible ground motions that can occur during the service life of the structure (i.e. the design earthquake). There are no acceleration records from earthquakes of magnitude larger than 7·7 in California. In the study the response of the longitudinal direction on the building under more severe ground motions was evaluated with the use of two earthquake ground motions recorded in California on alluvium deposits which have a similar frequency content to that of recorded motions at the base of the building and with relatively high input energies in the vicinity of the fundamental period of the building. Although the selected ground motions may differ from ground motions at the base of the building during large magnitude earthquakes, they can provide insights on the weak points in the building and on the approximate amplitude of local demands in the building during severe earthquake ground motions.

The first ground motion is the north–south component of the Hollister record, obtained during the 17 October 1989 Loma Prieta earthquake ($M_s = 7{\cdot}1$) in the city of Hollister approximately 48 km (29·8 miles) southeast of the epicenter. The digitized record has a total duration of 40 s, a peak ground acceleration of 361·9 gals (0·37g), and a maximum incremental velocity (maximum change in velocity from a negative peak to a consecutive positive peak or from positive peak to a consecutive negative peak) of 121·6 cm/s (47·9 in/s). This ground motion is characterized by a low-frequency content and a long-duration acceleration pulse (lasting 0·7 s) at approximately the 8 s mark and another (lasting 1·0 s) at approximately the 12 s mark.

The second ground motion is the S50W component of the James Road record, obtained during the 15 October 1979 Imperial Valley earthquake ($M_L = 6{\cdot}5$) approximately 10 km (6·2 miles) north of the epicenter. The digitized record has a total duration of 39·3 s, a peak ground acceleration of 360·4 gals (0·37g), and

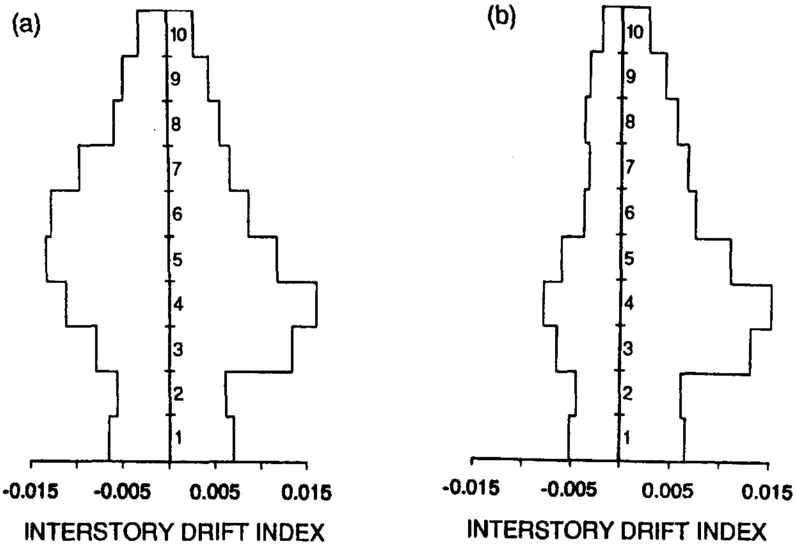

Figure 13. Envelopes of interstorey drift index computed through non-linear time-history analyses using: (a) The Hollister record; (b) The James Road record

Table VI. Summary of response parameters computed in non-linear time-history analyses

Response parameter	Hollister	James road
Max. base shear/W	0·23	0·22
Max. displacement (cm)	19·3	19·6
Max. IDI (†)	0·0161	0·0154
Min. base shear/W	− 0·23	− 0·18
Min. displacement (cm)	− 18·9	− 11·4
Min. IDI	− 0·0132	− 0·0078

Notes: (†) Interstorey Drift Index

a maximum incremental velocity of 160·0 cm/s (63·0 in/s). This ground motion is characterized by a long-duration acceleration pulse (lasting 1·6 s) at approximately the 6 s mark.

When subjected to the Hollister record the maximum roof displacement and maximum normalized base shear are 19·3 cm and 0·23, respectively. Both response parameters occur as a result of the long-duration acceleration pulse which occurs at the 8 s mark. Envelopes of interstorey drift index are shown in Figure 13(a). It can be seen that deformations concentrate from the 3rd to 7th stories. The maximum storey displacement ductility ratio is 3·15 which occurs at the fourth storey, which corresponds to an interstorey drift index of 0·016. Columns in this storey undergo four yield reversals and a maximum rotation of 0·02 rad.

When subjected to the James Road record the maximum response of the building is produced as a result of the long-duration acceleration pulse occurring at the 6 s mark. The maximum roof displacement and maximum normalized base shear are 19·6 cm and 0·22, respectively. Envelopes of interstorey drift index are shown in Figure 13(b). Similarly as in the case of the Hollister ground motion, inelastic deformations concentrate between the 3rd and 6th stories. The maximum computed storey displacement ductility ratio is 3·03 which again occurs in the fourth storey, corresponding to an interstorey drift index of 0·015. Columns in this storey only undergo two inelastic excursions (one in each direction). A summary of response parameters

Table VII. Summary of storey displacement ductility ratios computed using the Hollister and James Road records

	Storey displacement ductility demand	
Storey	Hollister	James road
10	0·59	0·51
9	0·92	0·81
8	1·10	1·06
7	1·77	1·32
6	2·36	1·52
5	2·61	2·23
4	3·15	3·03
3	2·64	2·61
2	0·96	0·99
1	0·88	0·83

and maximum storey displacement ductility demands for both ground motions are presented in Tables VI and VII, respectively.

For these levels of inelastic deformation, significant damage and strength degradation could occur between the third and sixth stories, particularly in the event of ground motions associated with large magnitude events (i.e., with longer durations) where the number of yield reversals is likely to be larger than those computed with the ground motions considered herein.

SUMMARY AND CONCLUSIONS

Results of analytical studies conducted on an instrumented ten-storey reinforced concrete building which experienced ground accelerations in excess of 0·6g during the 1987 Whittier-Narrows California earthquake and suffered only minimal damage have been presented.

A three-dimensional linear elastic model of the building calibrated using dynamic characteristics inferred acceleration time histories recorded in the building produced a very good correlation with the response measured during the Whittier-Narrows earthquake. The number of modes required to capture the response of the building varied with the response parameter. While maximum global displacements were captured adequately using only the first mode in each direction of the building, the estimation of acceleration time histories required the inclusion of all modes with frequencies smaller than that corresponding to the second translational mode of the transverse direction.

Significant overstrengths (strengths beyond the minimum required by the code) were computed in the building. For the longitudinal direction the ratios between the maximum lateral strength capacity under static loads and the code (ultimate strength) design base shear were 4·23 and 5·0 for the triangular and uniform lateral loading patterns, respectively. These overstrengths were obtained assuming that the structure would be able to develop a global displacement ductility ratio of about 2·4 and a local (storey) ductility ratio of about 3·7, although it is doubtful that the existing detailing of the reinforcement would allow this to be developed, particularly under cyclic loading.

For the transverse direction a larger overstrength was computed. In this case, the ratios between the maximum lateral strength capacity under static loads and the code (ultimate strength) design base shear were 5·75 and 6·99 for the triangular and uniform lateral loading patterns, respectively. These overstrengths were obtained assuming that the shear walls could develop a global displacement ductility ratio of about 3. It is doubtful that the detailing of the coupling girders and walls could allow such a high global displacement ductility ratio to develop without significant strength degradation.

These overstrengths were instrumental in avoiding damage in the longitudinal direction and the occurrence of only minimal damage in the transverse direction. Had the building had only the minimum lateral strength required by the code the damage would have been significantly larger. Computed interstorey drifts in the building when subjected to ground motions recorded in the basement during the Whittier-Narrows earthquake explain the absence of more damage in the building.

In spite of having peak ground accelerations significantly smaller than those recorded in the basement of the building during the Whittier-Narrows earthquake, the Hollister and James Road records produced large inelastic deformations along the 3rd to 7th stories.

The estimation of seismic demands on new and existing structures depends on the consideration of the real lateral strength and stiffness of the building. There is a need to study the overstrength of existing buildings according to their structural system and their height. It is recommended than in order to increase the reliability of current building code seismic provisions, reductions in the required design lateral strength should explicitly consider probable values of the lateral overstrength. Recent studies have suggested the use of strength reduction factors which are the product of reductions due to non-linear behaviour times an overstrength factor.[18–20]

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